22. Stagnation Pressure Failure of Spillway Chutes

Key Concepts

Description of Potential Failure Mode

Stagnation pressure related spillway failures can occur as a result of water flowing into cracks and joints during spillway releases. If water entering a joint or a crack reaches the foundation, failure can result from excessive pressure and/or flow into the foundation. If no drainage exists, or if the drainage is inadequate, and the slab is insufficiently tied down, the build-up of hydrodynamic pressure under a concrete slab can cause hydraulic jacking. If drainage paths are available, but are not adequately filtered, erosion of foundation material is possible and structural collapse may occur. Figure 22-1 depicts the development of stagnation pressures under a spillway chute slab.

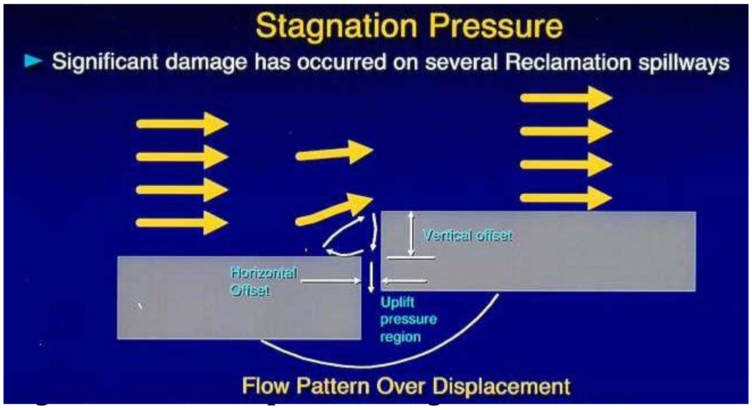

Figure 22-1 Stagnation Pressure Development

Condition of Concrete in Spillway Chute

Cracks, offsets and/or open joints in chute slabs and the lower portions of chute walls exposed to flow, and especially vertical offsets into the flow allow for this failure mode to initiate. Concrete deterioration in the form of delamination, alkali-silica reaction, freeze-thaw damage and sulfate attack can exacerbate this failure mode by initiating cracks, opening cracks and joints in the chute concrete, creating offsets into the flow, and causing separation of the chute from the supporting foundation.

Defensive Design Measures

Defensive design measures can prevent the failure mode from initiating or can prevent the failure mode from developing. Defensive design measures include the following (listed in order of decreasing effectiveness): waterstops (can block path for water flow through joints in slabs); transverse cutoffs (prevents vertical offsets at transverse joints and limits path for water from inside of chute to foundation); longitudinal reinforcement/dowels across chute floor joints (minimizes width of cracks and openings at joints and may prevent offsets); anchor bars (provides resistance to uplift pressures lifting slabs off foundation); filtered underdrains (relieves uplift pressures that can be generated under slabs - filtering prevents movement of foundation materials into drainage system and initiation of foundation erosion); and insulation (which insulates the drainage system and prevents it from freezing, and also prevents frost heave locally). An absence of defensive design measures can allow initiation and progression of this failure mode. Figure 22-2 shows these defensive design measures.

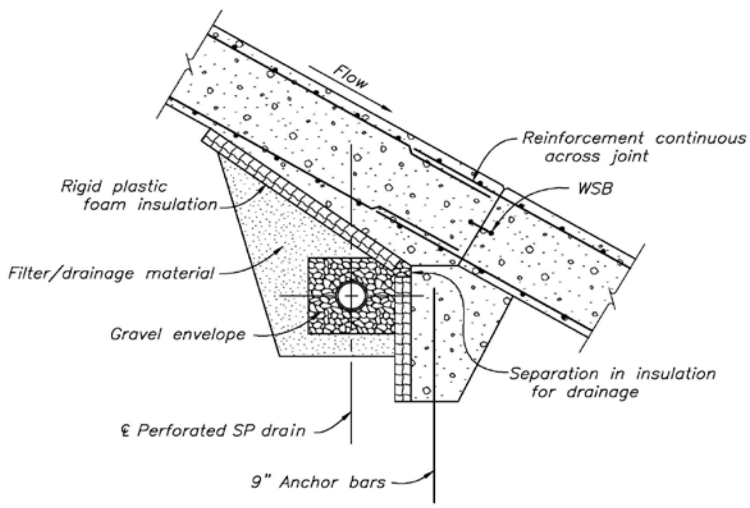

Figure 22-2 – Defensive Design Measures

Flood Routing Results/Flood Frequency

Routings of specific frequency floods provides discharges and discharge durations for a flood with a given return period. This information can be used to generate specific discharge level probabilities.

Spillway Discharges (Depths, Velocities, and Durations)

Water surface profiles can be calculated for discharges that are obtained from the routings of frequency floods. The water surface profiles will provide depths of flow and velocities at selected stations along the spillway chute. The flow velocity at joints and cracks in the spillway chute will help determine the magnitude of pressure that can be generated underneath the chute slabs and the volume of discharge that can be introduced through the crack or joint. Flood routings will provide information on the duration of certain discharge levels. If durations of spillway flows are limited, failure of the spillway chute may initiate but may not have time to fully develop into a breach of the reservoir.

Erodibility of Foundation Materials

Soil foundations are generally more erodible than rock foundations. If erosion of the foundation materials initiates and progresses, this could lead to undermining of the chute slab foundation and collapse of the chute slab. If a chute slab fails due to stagnation pressures and hydraulic jacking of the slab, headcutting and upstream progression of erosion will be a function of the erodibilty of the foundation materials (see section on Erosion of Rock and Soil). If the foundation consists of competent rock, upstream progression of erosion may be limited.

Spillway Configuration

Uncontrolled spillways can not be regulated and provide little or no opportunity to reduce discharges and control flows should problems develop during flood releases. Gated spillways may allow the opportunity to reduce flows (assuming that there is adequate surcharge storage space to allow this to happen without risking an overtopping failure of the dam) and slow down or arrest failure of the entire spillway if this failure mode is in progress.

Event Tree

Figure 22-3 is an example of an event tree for this potential failure mode (only one branch shown completely). The event tree consists of a number of events that lead from initiation, through progression, to full development of the failure mode. The first node represents whether Unfavorable Joints or Cracks Exist. The second node represents the starting reservoir water surface elevation and the third node represents flood load ranges. The combination of these two nodes represents the combined load probability and determines the range of spillway discharges that apply to each branch. The remaining nodes in the event tree represent the conditional probability of failure given the load. The remaining nodes include the following: 4) Spillway Flows Capable of Lifting Slabs or Eroding Foundation Materials Occur; 5) Defensive Design Measures are Inadequate and Stagnation Pressures Initiate Failure; 6) Unsuccessful Intervention; 7) Headcutting Initiates; and 8) Headcutting Progresses until a Breach Forms. Node 5 can actually have two variations – the first involving the initiation of hydraulic jacking and the second involving the initiation of foundation erosion, leading to structural collapse. Since the flood load range probability is typically dominated by the lower end of the range, the failure probability should also be weighted toward the lower end of the range. Refer also to the section on Event Trees for other event tree considerations. With the tools currently available, the estimates for most nodes on the event tree must by necessity be subjective (see section on Subjective Probability and Expert Elicitation).

Flood Studies/Flood Routing Analyses/Water Surface Profiles

A flood frequency study, along with the development of frequency hydrographs is required to fully evaluate this potential failure mode. Flood hydrographs should include a range of floods from the point where spillway releases become significant (for some spillways, very low flows may be significant) up to the Probable Maximum Flood (PMF).

A flood routing study is then conducted in which the frequency floods are routed and spillway discharges and durations determined for each flood event. If the starting reservoir water surface elevation is likely to vary (based on historical reservoir elevations), the routings should be performed with a number of different starting reservoir water surface elevations.

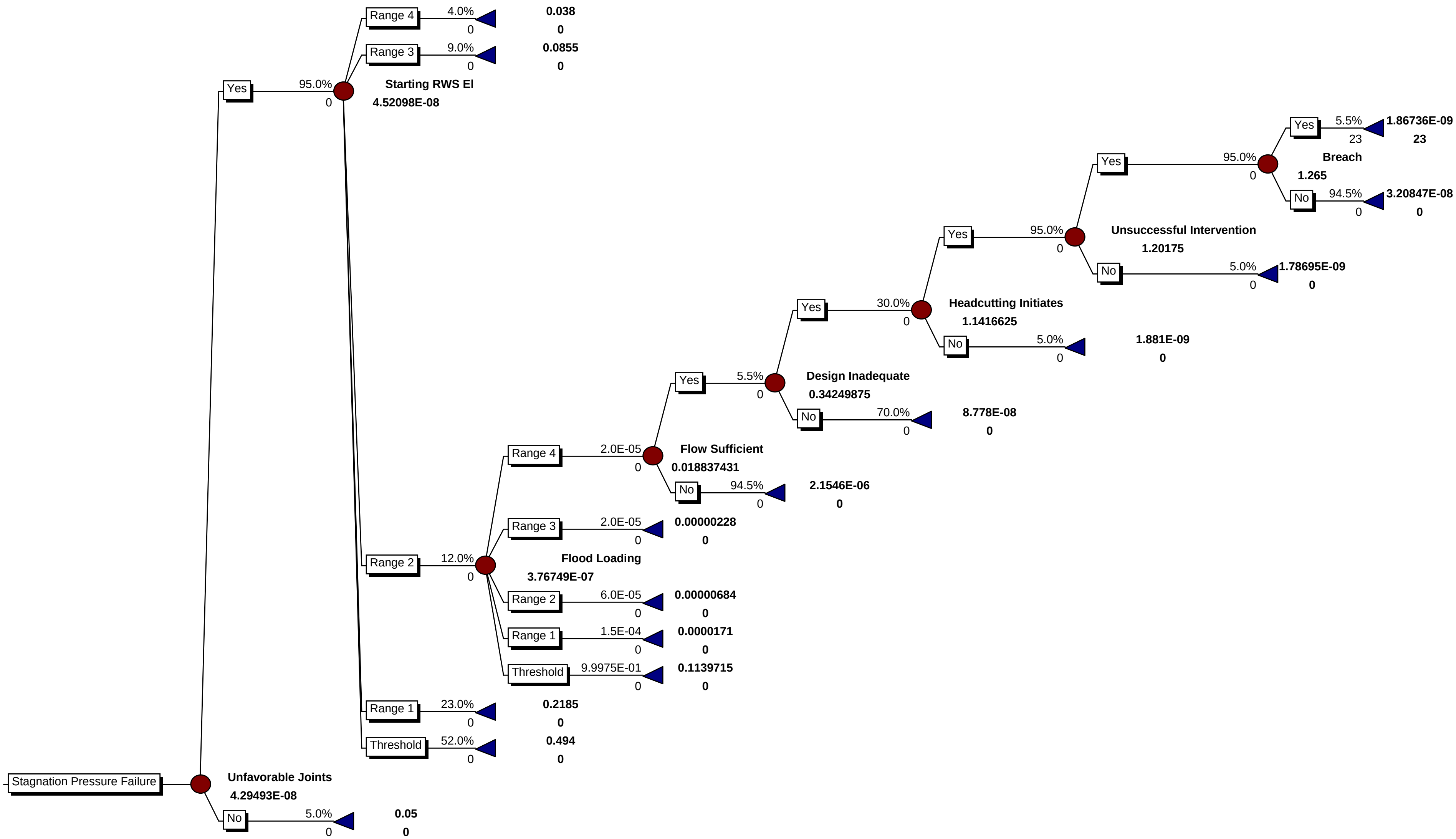

Figure 22-3 – Example Event Tree

Water surface profiles are then generated, using spillway discharge information from the frequency flood routings. For a given discharge and starting water depth at the spillway crest, flow depths and velocities can be determined at key stations along the spillway chute. This information along with information on offsets and joint/crack openings in the spillway chute can be used to estimate probabilities for the initiation of this potential failure mode.

Spillway Inspections

It is generally better to inspect the spillway chute for joints and cracks prior to assessing the risk. Locations where these features exist, particularly those that project into the flow, should be noted so that flows at that location can be studied in detail. A 1/8-inch offset may not be noticeable at a distance, but can produce significant stagnation pressures during the right flow conditions. However, it is not always practical to inspect a spillway chute, particularly for a screening level analysis. When available, design and construction details can be studied. When a spillway is suspected of having unfavorable conditions, it may be reasonable to assess the risks both with and without assuming that these conditions exist during a screening level analysis. This will indicate how critical it might be to acquire this information.

When a spillway is inspected, the inspection team should have knowledge of the design and construction of the features. A number of defensive measures were listed above. Prior knowledge of defensive measures that are absent will help determine how a potential failure mode may develop. This will help in identifying areas where unfavorable conditions exist during the inspection. For example, if all joints are keyed and contain waterstops, the inspection team may only be concerned with cracks or joints where damage or deterioration extends beyond the waterstop. Delamination is not always apparent during a visual inspection. Rapping the concrete surface with a hammer or other object may produce a hollow sound if delamination is present. A delaminated surface may be eroded during high flows, creating a condition where the remaining concrete projects into the flow at a joint. Heave or settlement of the spillway chute slabs may produce offsets that are difficult to detect. Rapping the concrete surface may identify hollow sounding areas which indicate a gap exists beneath the slab, perhaps due to erosion of soil into the drains, and soil deposits in the drain outfalls may indicate erosion is progressing. If unfiltered drains are suspected, ongoing erosion of foundation material through the drains could eventually result in offsets due to settlement. If the spillway has operated in the past, signs of settlement may already be present. Drains can be inspected to evaluate whether foundation materials have been moved into the drains. This can be done by inspecting the drain outfalls and by inspecting the underdrains with an ROV camera.

A detailed inspection of the spillway will likely result in specific areas of concern related to stagnation pressures. These areas may get special attention during a failure mode analysis or a risk analysis. Periodic inspections should focus attention on areas where the risk resulting from potentially offset joints is greatest.

Unfavorable Joints or Cracks Exist

There are generally two conditions that must be present for a stagnation pressure failure mode to initiate. First, there needs to be a crack or joint where flow and/or pressure can enter and access the foundation or interface. Secondly, the joint or crack also needs to be offset into the flow such that significant stagnation pressures can develop. Offsets away from the flow tend to aspirate through the joint or crack – lowering the pressure and pulling drainage back into the spillway chute. Longitudinal cracks that are open will allow seepage flows from the spillway chute into the foundation. These seepage flows could initiate foundation erosion when the spillway is operating, but will be of a lower magnitude than what would occur at a transverse joint or crack with an offset into the flow.

Structural damage at the joints can take several forms. This includes delamination and spalling. In addition, there may be offsets related to foundation heave or settlement. Delamination typically occurs near the surface, above the top layer of reinforcement. If a waterstop is installed below the delamination, a problem may not develop unless the concrete further deteriorates, compromising the waterstop. It is suspected that the combination of high surface temperatures and a plane of weakness (i.e. layer of rebar) below the surface can result in delamination. The concrete surfaces generally reach higher temperatures when the surfaces face direct sunlight. Expansion of the concrete on the surface can produce differential stresses throughout the concrete depth. There may be a “splitting” tensile force parallel to the surface. Reinforcement parallel to the surface creates a plane of weakness due to the reduced concrete area. Damage may be most significant in portions of the chute that have the greatest exposure to direct sunlight.

Spalling at joints may be caused by freeze-thaw damage, alkali-silica reaction (ASR), or poor concrete consolidation. When spalling occurs, a deep localized offset will be present. If deep enough, spalling may compromise other defensive measures, such as watertops or keys.

Starting Reservoir Water Surface Elevation

Starting reservoir water surface elevation ranges are used as nodes in the event tree if varying this parameter made a significant difference in the flood routing results. If this parameter is significant, the reservoir load ranges are typically chosen to represent a reasonable breakdown of the starting reservoir range from the normal water surface to an elevation representing the lower limit of what could typically occur before a major flood. This would typically result in several (maybe 3 to 4) reservoir load ranges. Historical reservoir elevation data can be used to generate the probability of the reservoir being within the chosen reservoir ranges, as described in the section on Reservoir Level Exceedance Curves.

Equally important may be reservoir operating criteria. The reservoir may have a range or ranges of water surface elevations where outflows are restricted. Restricted outflows can minimize the effectiveness of lower starting reservoirs.

Flood Load Ranges

Flood load ranges are typically chosen to provide a reasonable breakdown of the flood loads from the maximum flood routed (with the Probable Maximum Flood (PMF) representing the maximum flood that would be considered) to a threshold flood where the spillway discharges are at a level where failure due to stagnation pressures is judged to be remote. This would typically result in several (maybe 3 to 6) flood load ranges. There may be flood flow ranges where stagnation pressure potential is relatively constant. This is because once a critical velocity is reached in the spillway chute (where hydraulic jacking pressures exceed resisting forces), the potential for initial failure of the chute will significantly increase, but remain relatively constant (or increase slowly) for higher velocities. The flood duration following this critical condition becomes an important factor. The flood load ranges should be carefully chosen, and may not be the same ranges used to evaluate failure potential of the dam due to hydrologic loading. Flood frequency curves (or hydrologic hazard curves) are used to generate the probability distributions for the flood load ranges, as described in the section on Hydrologic Hazard Analysis.

Spillway Flows Capable of Lifting Slabs or Eroding Foundation Materials Occur

Having established that there are, or may be unfavorable conditions at joints or cracks, the likelihood of flows that can initiate potential failure can be estimated. Model tests for offsets into the flow as small as 1/8-inch, with gaps as small as 1/8-inch indicate that significant pressures and or flow can develop (Figures 22-4, 22-5, 22-6, 22-7 and 22-8, Reclamation, 2007). It should be noted that some of these tests were conducted with a sealed water vessel beneath the “slab”, where all hydrodynamic pressures were transmitted to this system (see Figure 22-4). Head loss that would occur as the water traveled through cracks, foundation materials, or drains was not modeled or captured. These results are, therefore, quite conservative and should be used with caution. A second set of tests were conducted, in which the water vessel beneath the “slab” was opened or “vented” by allowing drainage out of the cavity (see Figures 22-5 and 22-6). These tests allowed flow through the system, and uplift pressures on the “slab” were measured as well as flow through the joint. Venting of the cavity produced a reduction in uplift pressure for all test configurations. However, it should be noted that the drainage flow rates were a function of the losses within the test system. These include the crack entrance losses and losses within the piping and valve that allowed water to flow out of the area beneath the “slab.” Therefore, if enough drainage was provided to accommodate all flow that tended to enter the crack, the uplift pressures could be even lower.

Figures 22-7 and 22-8 provide unit discharges for variable flow velocities, joint offsets and joint gaps. It should be noted that the discharges represented in Figures 22-7 and 22-8 are based on the pressure and drain conditions reflected in the companion curves provided in Figures 22-5 and 22-6. The unit discharges provide estimates of flow through the joint that are consistent with the uplift pressures shown in Figures 22-5 and 22-6, since once again the flow was controlled by the valve used to model the vent in the experiments. The unit discharge values can be used to help assess whether the spillway underdrain system capacity is adequate to reduce uplift pressures to those levels indicated in Figures 22-5 and 22-6. An interesting result of the tests is that the test configurations with the smallest joint gap (see Figure 22-7) resulted in more flow through the joint as compared to test configurations with larger gaps (see Figure 22-8). A smaller gap also results in higher uplift pressure. The postulated reason for this is that a recirculation zone is created at the point of the gap entrance that is more effective in blocking flow and transmission of stagnation pressure at larger gaps. The details of the joint were also varied in the studies. Sharp edged joints were tested as well as joints with chamfered and rounded corners. The chamfered and rounded corners with small gaps performed in a similar manner to sharp edged joints with wider gaps.

There have been no specific tests for a joint that is not displaced into the flow, or with smaller gaps; however, it would seem possible that some flow and pressure could develop without an offset. Note that the stagnation line (black) in Figure 22-4 represents an upper bound or theoretical pressure that could be developed by converting the velocity head entirely to pressure. Additional conditions have been evaluated in the model tests, and are presented in the Reclamation (2007). After flow rates are determined for various flood frequencies, water surface profiles can be developed to determine flow depth and velocity. Both may be important factors. In general, studies have indicated that pressures and flows into offset joints and cracks increase with flow velocity (Figures 22-4, 22-5 and 22-6). For a given flow, there may be portions of the spillway chute that experience velocities that are high enough to cause damage, while other portions do not. If the portions of the spillway chute experiencing the potentially damaging velocities are not prone to failure because they have adequate defensive measures, lack unfavorable cracks or joints, and/or lack of offsets into the flow, failure is not likely to initiate. As flows increase, other portions of the chute without adequate protection may experience conditions that can initiate failure. Therefore, there may be a specific flow for different sections of a spillway chute that will represent an initiating failure condition.

The starting water surface and load range combinations (nodes described above) should be adjusted based on the flows that are considered significant in the spillway being evaluated. The flows in the lower portion of the chute that are significant may begin at a certain discharge; while in the upper chute, significant flows begin at a different discharge. Both of these flows may be the lower end of different ranges being considered.

Depth of flow may be important when there is an increasing offset between two wall segments that increases with height, or where damage has occurred above the chute invert. Wall offsets are often observed near spillway crest structures where a wall transitions from a high counterforted wall to a lower cantilevered wall.

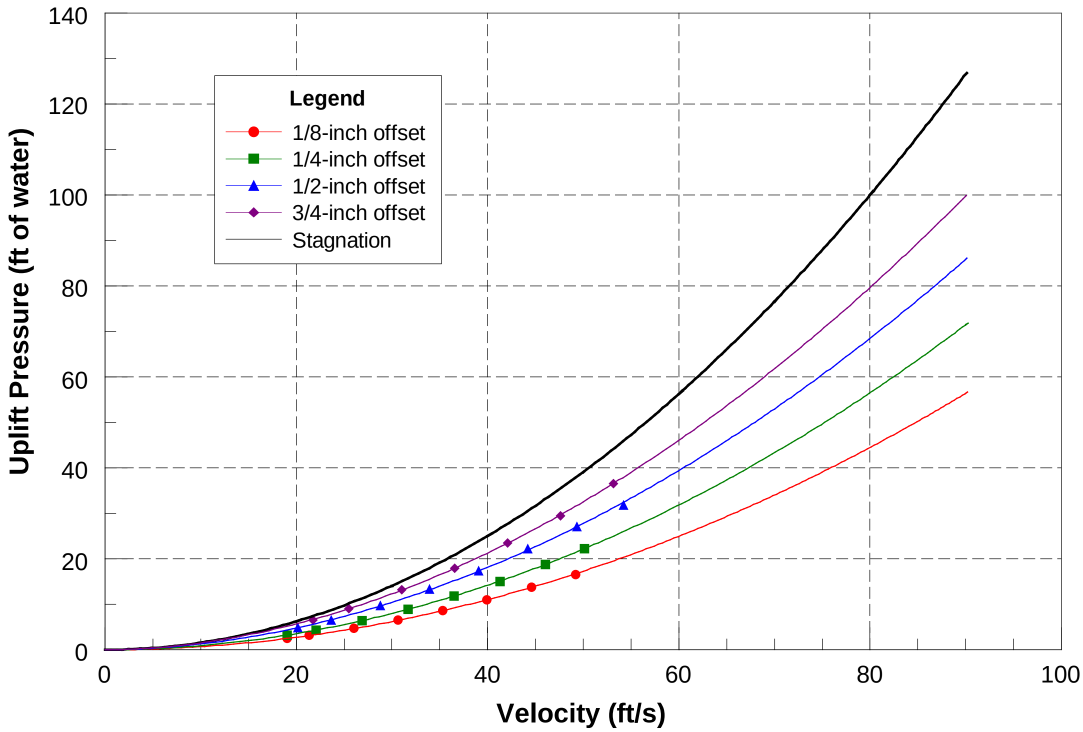

Figure 22-4 - Mean uplift pressure, sharp-edged geometry, sealed cavity, 1/8-inch gap.

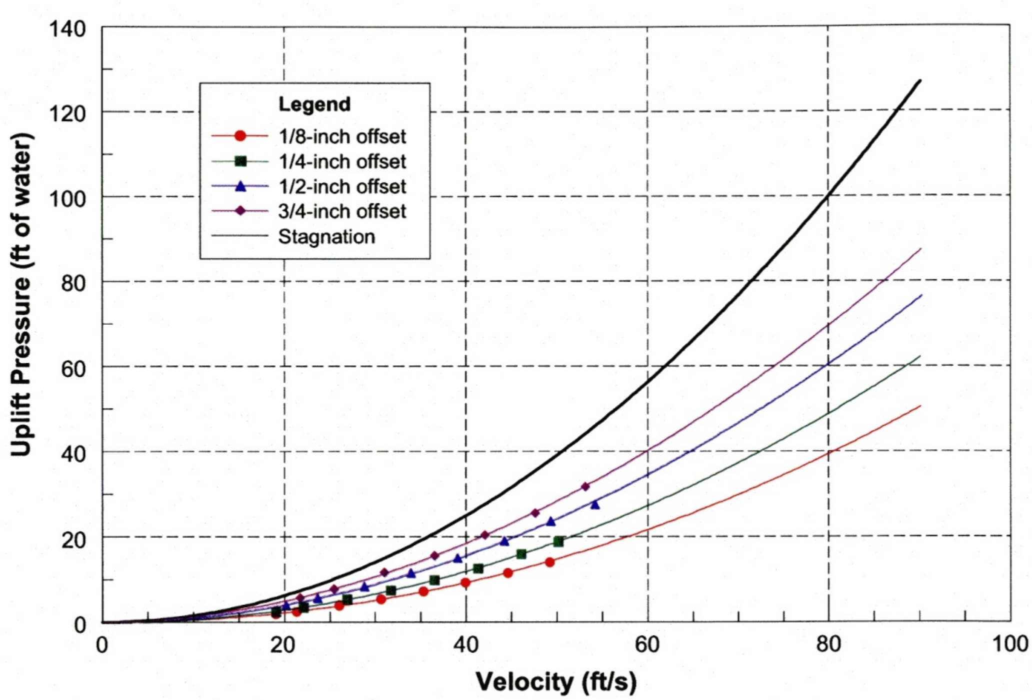

Figure 22-5 – Mean uplift pressure, sharp-edged geometry, vented cavity, 1/8-inch gap.

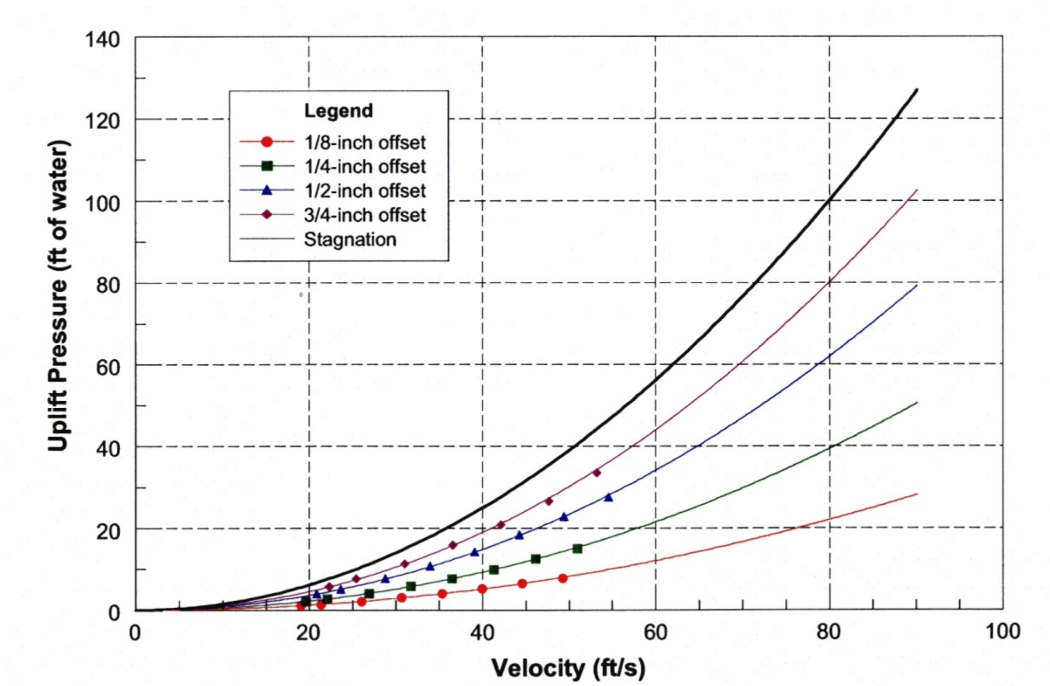

Figure 22-6 – Mean uplift pressure, sharp-edged geometry, vented cavity, 1/2-inch gap.

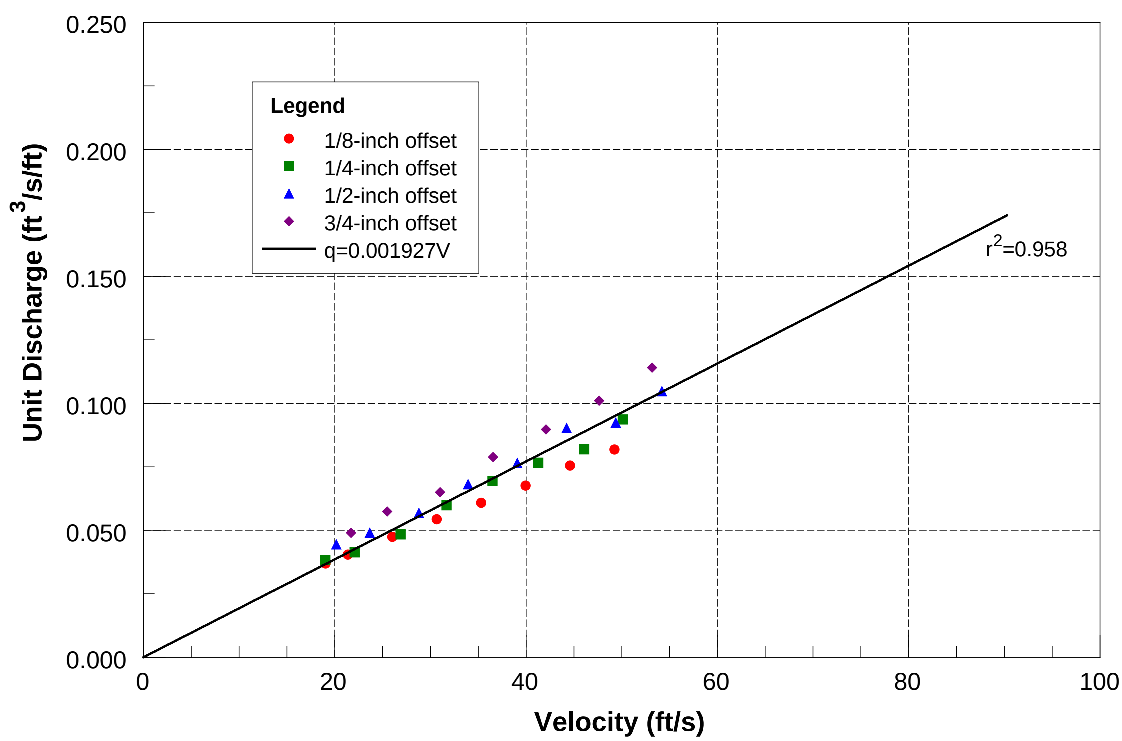

Figure 22-7 – Unit discharge for joint/crack, sharp-edged geometry, 1/8-inch gap.

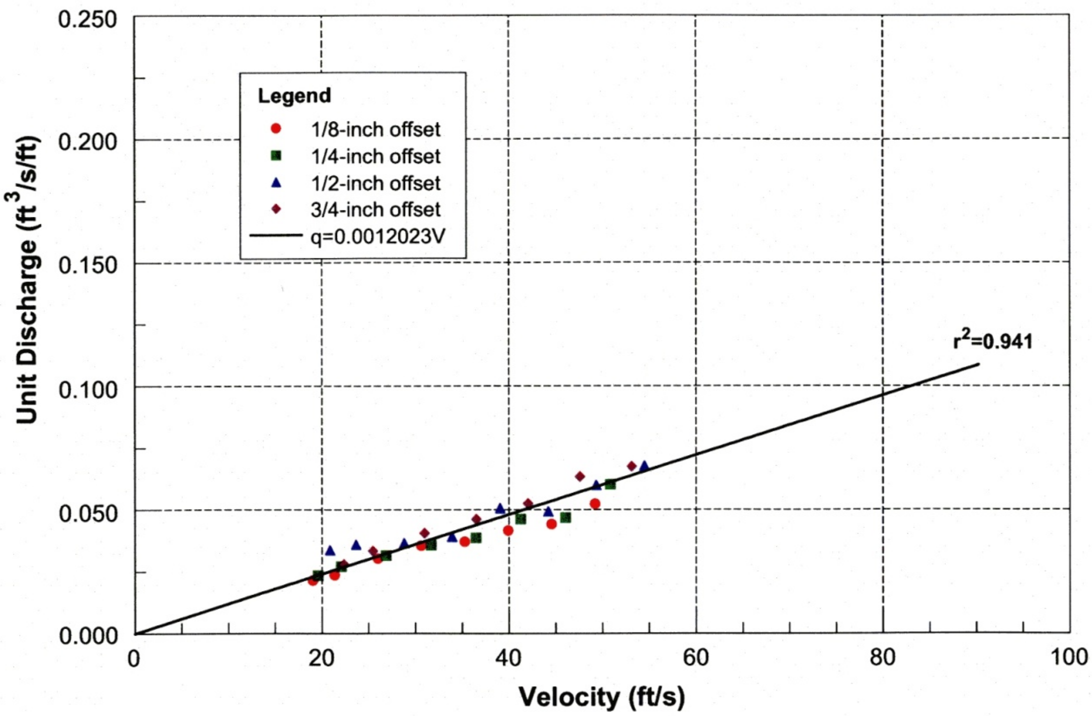

Figure 22-8 – Unit discharge for joint/crack, sharp-edged geometry, 1/2-inch gap.

Defensive Design Measures are Inadequate and Stagnation Pressures Initiate Failure

Once it has been established that unfavorable joint or crack conditions exist, and that flows capable of initiating chute failure (either through foundation erosion and eventual structural collapse, or hydraulic jacking) can occur, the adequacy of defensive measures included in the design and construction of the spillway should be evaluated. In evaluating the likelihood that defensive measures are inadequate, it is important to consider the combination of features. For this "event" defensive measures are generally those included in the design of joints in the concrete chute and features of the foundation. Joint details may include intentional offsets away from the flow (the upstream side is higher than the downstream side), joint details that prevent the downstream portion of the chute slab from displacing upwards relative to the upstream portion of the slab (these first two details will prevent offsets into the flow from developing or worsening if uplift pressures develop during a spill event), shear keys across slab joints, continuous reinforcement or dowels, anchor bars, drainage, and waterstops. Some defensive design features that are effective for joints will not be effective for cracks that develop in concrete slabs. Waterstops will not be provided at cracks and joint details that prevent the chute slab downstream of a transverse joint from being offset upwards relative to the slab upstream of the joint will not exist at a crack location.

Many older spillways have a single layer of reinforcement. Even if the reinforcement were continuous across the joints, it may not be adequate to prevent offsets into the flow, or to keep the slabs acting together if uplift initiates. These lightly reinforced structures may also be deficient in terms of crack control. If waterstops are not flexible (which may be the case for metal waterstop or old plastic waterstops), differential movement at joints may fail the waterstop. Waterstops may also be undersized (inadequate embedment) based on aggregate size, or improperly embedded (not enough cover or concrete consolidation). Waterstops in older spillways (if included at all) may have been placed at the crest structure and/or stilling basin, but may not be present for the entire length of the chute.

Joint details may provide significant defense, especially those that prevent lifting of the downstream slab relative to the upstream slab. An effective joint detail may include a cutoff into the foundation below the joint that supports both the upstream and downstream portion of the chute slab (as shown in Figure 22-2). Shear keys at transverse floor joints may also be effective in preventing offsets at joints. However, many keys have a reduced thickness as compared to the slab, and shear strength may be an issue for some load cases, such that shearing of the key between slabs under large depths of flow allows for differential movement.

Foundation drainage can be designed to relieve uplift pressures caused by normal groundwater conditions. However, excessive inflows from joints and cracks in the concrete may exceed drain capacity (see Figures 22-7 and 22-8). Foundation drainage in older spillways was often constructed without adequate filtering. Any water flowing through the foundation could carry foundation materials into these unfiltered drains, possibly resulting in erosion, formation of a void, and eventual structural collapse of both the drainage system and the concrete above. Spillway flows may not be necessary for this condition to develop if there is groundwater flow to the drain system. If this is occurring, the slabs may be more susceptible to settlement under flow conditions. Spillways on a rock foundation may include anchor bars to tie the foundation rock and chute slab together.

If it is found that defensive measures may not be adequate given the spillway flow and potential for significant stagnation pressures to develop, there are two modes of failure to consider. At this point in an event tree, there may be two failure branches: a) Hydraulic jacking initiates; or b) Foundation erosion (leading to structural collapse) initiates. One or both of these failure modes may be possible, and if deemed possible, the likelihood of each initiating needs to be estimated. Figures 22-4, 22-5, 22-6, 22-7 and 22-8 provide a basis for estimating failure potential (data for a variety of other joint/crack conditions are provided in Reclamation, 2007). For the branch related to hydraulic jacking, estimates can be made of the uplift forces under the slab (from Figures 22-4, 22-5 and 22-6) and a determination made as to whether this exceeds the resisting forces preventing slab jacking (weight of slab and water above the slab, anchor bar resistance, and structural resistance provide at the borders of the slab or reinforcement passing through the joints). If the chute slab is monolithic with the spillway chute walls or if reinforcement is provided across longitudinal joints in the chute slab, the walls and the surrounding backfill will likely be mobilized if uplift pressures develop. This will provide resistance against hydraulic jacking of the chute slabs. If this is the case, the moment and shear capacity of the slab will need to be checked to make sure these are not exceeded.

Some judgment is needed as to whether the results indicated in Figure 22-4 or 22-5 are more likely to occur. If drainage is provided, judgments will need to be made on the effectiveness of the drains in reducing uplift pressures. To aid in this judgment, comparisons can be made between the drain capacity and the potential discharge into cracks and joints indicated on Figures 22-7 and 22-8. However, it should be noted that these figures were developed by including a relatively small outlet drain in the model.

Thus, the flow rate was influenced by the capacity of the drain system. Figures 22-7 and 22-8 may be quite conservative if there is substantial drain capacity under the actual spillway slabs. If the drains become pressurized in order to pass the flow, they will have diminished capability to relieve uplift.

Judgments will also have to be made on the lateral extent of uplift pressure development. If the slab is expected to be well bonded to a relatively impervious foundation, the lateral extent may be limited. If there is evidence that the slab has been lifted off the foundation, or the foundation is permeable, the lateral extent may be extensive. If there is potential for the uplift pressure to extend a significant distance downstream where the invert is much lower, the uplift at this point may be higher than at the point of entry (if drainage from the lower location is impeded).

For the branch related to foundation erosion, judgments will have to be made as to whether an unfiltered exit exists (typically into an underdrain system), and whether the foundation materials are erodible. Evidence of foundation erosion identified during a site inspection may be useful in evaluating this branch. Duration of flows in the foundation may be a significant factor because enough material would need to be removed to cause the slab to shift or collapse. Minor settlement during this process could cause a significant increase in stagnation related flows and/or pressures. Another consideration is if foundation erosion has occurred in the past. The chute may be on the verge of collapse, and just a little more erosion or depth of flow could cause collapse. If this is the case, collapse is more likely than conditions where the foundation is still intact.

Unsuccessful Intervention

Once this failure mode initiates, successful intervention would prevent the failure mode from fully developing. The likelihood of this needs to be estimated based on the specific flows and conditions of each branch. One obvious form of intervention for a gated spillway is to close the gates. While this may prevent failure of the spillway, it could lead to other problems such as high reservoir loading on the dam or even dam overtopping. Therefore, closing gates may not be a practical solution for large floods, but may be possible for smaller floods that can be stored in the reservoir. Other forms of intervention may be possible, including diverting flows away from the failed section of the spillway. Interventions may also include armoring the failed spillway section, temporary closure of gates to construct a temporary fix, using an emergency spillway or outlet, or constructing a temporary spillway cut in a benign saddle or other area.

Headcutting Initiates

If intervention is not successful, foundation erosion initiating at the failed chute section could lead to headcutting upstream, and the likelihood of this needs to be estimated. As the first section of spillway fails, it exposes the foundation to full spillway flow. Whether headcutting initiates is primarily dependent on the erosion rate of the foundation and whether any initiation points, such as slope changes from flat to steep or changes in geology, exist in the profile. In general, rock foundations may take longer and require higher energy flows to initiate erosion than soil foundations. Hard bedding controlled "slabs" of rock beneath and parallel to the chute may be quite resistant to headcut initiation. Soil and rock properties play an important role in the initiation of erosion (see section on Erosion of Rock and Soil).

Headcutting Progresses until a Breach Forms

Given that a headcutting failure initiates, it could progress upstream to the reservoir. The duration of the flood flows may be critical to the likelihood of formation of a full reservoir breach. If the spillway foundation is somewhat erosion resistant, the headcutting may not reach the reservoir before the flood is over. In highly erodible foundations, the reservoir may be breached a short time after the headcutting is initiated. Some spillway crest structures may be founded on rock, or have cutoffs to rock. This would delay failure of the crest. Deep cutoffs beneath the chute may also prolong the headcutting advancement. Spillways adjacent to embankment dams may carry the added threat of erosion to the embankment (if headcutting leads to loss of spillway walls or water coming in contact with the embankment) leading to a breach once the chute walls fail.

Headcutting is typically considered a progressive failure. Once the first chute section fails and headcutting initiates, erosion would progress upstream under the upstream chute section. The upstream chute section may cantilever over the erosion hole until it becomes unstable and collapses. Erosion progresses under the next upstream section and the process is repeated until the spillway crest fails. All the while, additional erosion may be occurring downstream, but this may have little impact on the likelihood that the reservoir is breached.

Consequences

Loss of life for the stagnation pressure failure mode can be estimated from the predicted breach flows and the estimated population at risk that would be exposed to the breach outflows using the procedures outlined in the section on Consequences of Dam Failure. Incremental loss of life should be considered, which accounts for the fact that large spillway releases that may precede a breach of the reservoir through the spillway or breach area. In some cases significant portions of the downstream population may be affected by operational spillway flows prior to breach, which may force their evacuation prior to a breach of the reservoir, effectively reducing the population at risk (if they initially evacuate to an area outside the breach inundation zone). Large spillway releases will also create a heightened awareness for populations located along the river channel and may improve the flood severity understanding, should failure occur. Additionally, the dam and spillway will likely be closely observed in a flood situation, the failure mode will take some time to fully develop and if a reservoir breach develops it will likely be smaller than a full dam breach. All of these additional factors will increase warning time and improve the evacuation of downstream populations.

Accounting for Uncertainty

The method of accounting for uncertainty in the flood loading is described in the sections on Hydrologic Hazard Analysis and Event Trees. Typically, the reservoir elevation exceedance probabilities are taken directly from the historical reservoir operations data, directly, which do not account for uncertainty (see section on Reservoir Level Exceedance Curves). Uncertainty in the failure probability and consequences are accounted for by entering the event tree estimates as distributions (as describe above) rather than single point values. A “Monte-Carlo” simulation is then run to display the

uncertainty in the estimates, as described in the section on Combining and Portraying Risks.
There may be some uncertainty regarding spillway discharges for a given frequency flood, because of unpredictability of how the spillway will actually operate during a flood event. Spillway capacity may be limited due to debris plugging or malfunctioning of spillway gates during a flood event, which would reduce the spillway discharge for a given frequency flood. It is not recommended that concerns over reduced spillway capacity be considered for this potential failure mode, since in most cases the probability of these reductions are low and they are difficult to quantify. In addition, a reduction in spillway capacity would decrease the likelihood of failure under this mode, on the unconservative side.

There may be considerable uncertainty regarding the condition of the spillway chute, including whether open joints or cracks exist in the spillway chute (due to lack of a recent thorough examination of the chute concrete), whether waterstops are intact, whether the spillway chute slab is bonded to or in tight contact with the foundation and whether existing voids exist behind the spillway chute lining. These uncertainties need to be considered and incorporated into the risk analysis estimates. Where conditions are unknown and the assumptions are critical (such as whether drains are functioning or not), risk estimates can be made for the two extreme possibilities and the results evaluated. The difference in the two estimates may provide justification to initiate inspection, exploration, or testing programs. If drawings are not available that provide design details for a spillway chute being evaluated, the period in which the structure was designed and constructed can be used to make assumptions about what features were likely built, based on practices at the time.

Relevant Case Histories

Big Sandy Dam Spillway: June 1983

Big Sandy Dam is located on the Big Sandy Creek, 45 miles north of Rock Springs, Wyoming. The 85-foot high rolled earthfill embankment dam was completed in 1952. The spillway is located on the right abutment of the dam and consists of an uncontrolled concrete side-channel crest structure and a concrete chute and stilling basin. The spillway has a discharge capacity of 7350 ft^3/s at a reservoir water surface elevation 5.3 feet above the spillway crest elevation. The spillway is founded on thinly bedded to massive siltstone and sandstone. The foundation rock ranges from soft to moderately hard with joints that are primarily vertical, tight and healed to open and spaced from 1 foot to several feet apart. A zone in the foundation below the spillway inlet structure contains open joints and bedding planes, which allowed reservoir water to seep under the spillway chute floor. The spillway chute was designed with an underdrain system and anchor bars, but waterstops and continuous reinforcement were not provided across the contraction joints (Reclamation, 1987). Deterioration of the concrete slab occurred shortly after the dam was put into service. Cracking occurred in the chute slabs due to excessive water and ice pressures along the foundation-concrete slab interface and some of the slabs heaved and were displaced off the foundation, creating offsets into the flow. The spillway operated from 1957 to 1983 without incident, but a chute floor slab failed in June 1983, due to uplift pressures from flows of 400 ft^3/s (Hepler and Johnson, 1988). The failure did not progress beyond the spillway slab failure, primarily due to the erosion resistance of the underlying foundation relative to the energy of the spillway release flows.

Calculations were performed to confirm that the failure was the result of stagnation pressures being generated under the chute slab. The Big Sandy Spillway slab failed between stations 4 + 66.87 and 4 + 85.85, during spillway discharge of 400 ft^3/s. Failure was initiated by an offset into the flow at station 4 + 66.87 (depth of flow – 0.3 ft; velocity – 31 ft/s). Assuming a 1/8 inch open joint and a vertical offset of 1/2 inch and anchor bars only 50 percent effective, the calculations predicted the slab would fail. The calculations also showed that with anchor bars fully effective, the slab would not have failed. The uplift pressures assumed in the calculations were estimated from extrapolated laboratory tests (Hepler and Johnson, 1988). The analysis of the slab for uplift pressures evaluated a one foot wide strip of the chute slab between stations 4 + 66.87 and 4 + 85.85, assuming that the stagnation pressures would be constant over this area. From observations after the failure, it was observed that the anchor bars exposed beneath the slab were not coated with grout, indicating that the anchor bar capacity was not fully developed.

Hyrum Dam Spillway

Hyrum Dam is located on the Little Bear River, about 9 miles southwest of Logan, Utah. The 116-foot high zoned earthfill embankment dam was completed in 1935. The spillway is located about 900 feet from the dam on the right abutment and consists of a concrete lined inlet transition, a gated crest structure regulated by three 16-foot-wide by 12-foot-high radial gates, and a concrete lined spillway chute and stilling basin. The foundation of the spillway consists of Lake Bonneville sediments (described as clay and gravelly loam) to a depth of about 90 feet below the spillway crest. The spillway chute was designed with an underdrain system (although a filter was not provided between the gravel drain envelope and the fine-grained foundation material. The spillway chute was constructed with a single layer of reinforcement that is not continuous across the joints. Waterstops were not provided at the joints. The spillway had significant problems associated with cracking and slab movement. Long horizontal cracks developed in the sides of the trapezoidal spillway chute, and bulging of the lining was noticeable. In 1980, an inspection revealed water spurting through a crack in the left chute wall (indicating water pressure behind the wall) and open horizontal cracks. In 2003, ground penetrating radar, drilling and closed circuit television examination of the spillway underdrains and drill holes were used to identify voids underneath the spillway chute. A continuous channel, over two feet deep in places was identified beneath the steeper portion of the chute. The erosion that occurred in the spillway foundation was attributed to the introduction of flows through the cracks and joints in the slab and piping of foundation materials into the unfiltered drainage system (Reclamation, 2005).

Exercise

Consider a spillway with a concrete lined chute. The chute slab is 12 inches thick (measured normal to the slope) and is founded on a non-erodible rock foundation. The foundation rock is jointed but has low permeability. An underdrain system was provided, but it is plugged and no longer functions. Neither continuous reinforcement nor waterstops are provided across the chute slab joints. Anchor bars, consisting of No. 9, 40 $kips/in^2$ steel, on 10 foot centers (each way), attach the chute slab to the foundation. A recent inspection revealed tight joints in the spillway chute slab, with no offsets, except for the transverse joint at Station 17+04. Due to freeze-thaw damage to the concrete, a large concrete spall has created a 1/4-inch offset into the flow, across most of the chute width at this location. The joint at Station 17+04 has also opened laterally and averages about 1/8 in wide. The information in Table 22-1 was extracted from a water surface profile study:

Table 22-1 – Flow Depths and Velocities at Spillway Station 17+04			
Frequency Flood, yr	Spillway Discharge, ft^3/s*	Flow Depth, Normal to the Slope, ft	Flow Velocity, ft/s
1000	3500	2.5	35
10,000	8300	4.6	42
100,000	17,800	8.1	47
1,000,000	25,300	10.5	50

* Spillway discharges did not change appreciably with a variable starting water surface elevation.

Estimate the expected annual probability of the initiation of chute slab jacking due to high stagnation pressures.

References

1. Bureau of Reclamation, *Hydraulic Design of Stilling Basins and Energy Dissipators, Engineering Monograph No. 25*, Bureau of Reclamation, Denver, Colorado, Revised 1978.

2. Bureau of Reclamation, *Design Summary, Big Sandy Dam Safety Spillway Modification, Eden Project, Wyoming*, March 1987.

3. Bureau of Reclamation, “Hyrum Dam Issue Evaluation Hydrologic Risk Analysis, Technical Memorandum No. HYM-8130-IE-2005-2,” Technical Service Center, Denver CO, May 2005.

4. Bureau of Reclamation, *Uplift and Crack Flow Resulting from High Velocity Discharges over Open Offset Joints, Report DSO-07-07*, Technical Service Center, Denver CO, December 2007.

5. Hepler, Thomas E., and Johnson, Perry L., “Analysis of Spillway Failures Caused by Uplift Pressure,” Proceedings of National Conference on Hydraulic Engineering and International Symposium on Model-Prototype Correlations, ASCE, August 8-12, 1988.

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